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# Influence of soil liquefaction on dynamic response of structure on pile foundation

Shin'ichiro Mori, Takaaki Ikeda, Yukio Takimoto, Masato Muto & Takayuki Tohaya
*TOBISHIMA Corporation, Tokyo, Japan*

ABSTRACT: The objective of this study is to evaluate the influence of liquefaction on the dynamic response of structure on pile foundation during an earthquake. In this paper, the results of the analysis and the shaking table tests were compared, in order to judge the validity of the liquefaction analysis by the effective stress method for soil-structure system modeled as a simple spring-mass system. As a result, it was found that the proposed method is effective for competent assessment of the above mentioned influence. It is considered to be serviceable for design practices.

## 1. INTRODUCTION

The importance of assessing the impact of soil liquefaction on coastal reclaimed lands was recognized after the recent earthquakes in Japan and the United States, as well as the need for countermeasure against it, since many large-scale structures are planned to be constructed on such grounds in Japan.

The authors(1990) developed a practical method of dynamic response analysis of soil-structure system by effective stress method with the purpose of evaluating the influence of soil liquefaction on the dynamic response of structure on pile foundation during an earthquake. Further, in order to investigate the validity of these analysis methods, laboratory element tests for model parameters and shaking table tests were conducted using large shear box to simulate loose soil of a relative density of 50%.

## 2. METHOD OF ANALYSIS

### 2.1 Liquefaction analysis of soil

One-dimensional response analysis for level ground is done before analysis of coupled systems. The pore pressure generation model to be investigated here is the effective stress path model by Towhata and Ishihara(1980). The hysteretic Hardin-Drnevich(1972) Model (hereinafter referred to as H-D Model) was used for the shear stress-shear strain relationship.

### 2.2 Model of soil-structure system

Fig.1 shows proposed model for soil-structure system as a simple spring-mass system. This model consists of a structure-pile system, a near field system, and a free field system. The structure-pile system and the near field system are assumed to give the same horizontal displacements. These systems and the free field system are connected with interaction springs at the corresponding masses. The free field system and the near field system are lumped mass-shear spring models, in which the shear springs are the strain-dependent nonlinear springs of the above-mentioned hysteretic H-D model. Free field system is greater enough not to be affected by other 2 systems.

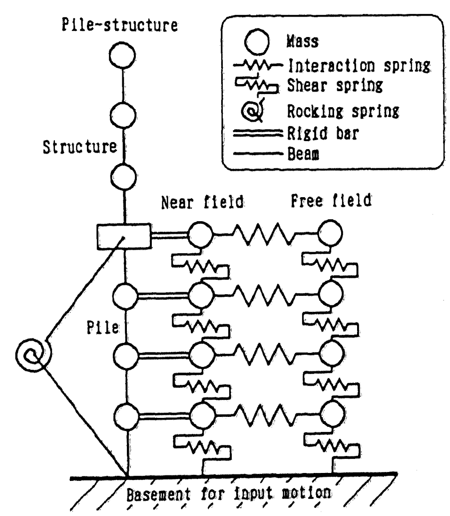


Fig.1 Proposed model for soil-structure system

In this model not only the acceleration input from the pile ends but also the action of earthquake force from the sides is taken into consideration. Here the interaction between the piles and the soil which is not affected by the structure (free field) is reflected as the force from the sides.

With a view to incorporate the results of the one dimensional analysis of liquefaction into the soil-related parts of these coupled models, the authors(1990) developed MEP2-L, a liquefaction analysis program for soil-structure system.

## 2.3 Nonlinearity of interaction spring and evaluation of soil liquefaction

Fig.2 shows the nonlinear characteristics of the interaction spring. Here, hysteretic H-D type nonlinear spring depending on deformation (relative horizontal displacement between the soil and the piles) is adopted, which is hysteretic H-D type or a hyperbolic relationship between the load and the relative displacement. The force-deformation relationship against irregular loads is supposed to follow Masing's rule (Hysteretic H-D type).

The change of stiffness and strength in nonlinearity depending on the confining pressure can be taken into consideration. It is done by incorporating the pore pressure time history obtained from the results of one dimensional liquefaction analysis into these parameters of nonlinear springs relating to the soil. Maximum spring stiffness Kmax and maximum spring force Fmax, the parameters of spring , vary to be proportional to the 0.5th power of the confining pressure and itself, respectively, corresponding to the excess pore pressure at the depth of interaction spring.

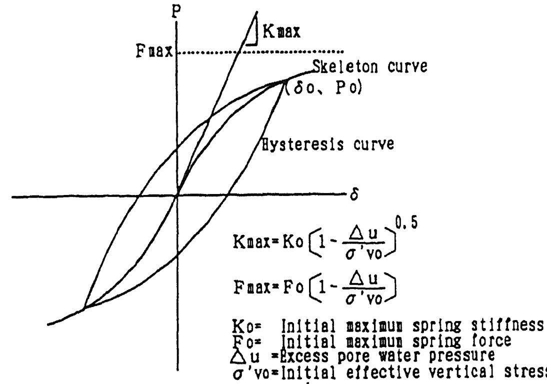

Fig.2 Characteristics of hyperbolic interaction spring

## 3. EXPERIMENTAL MODEL AND SHAKING TABLE TESTS

The similarity ratio of the length was 1/16, and that of the time was 1/8 according to the similarity law [Mori et al.(1990)].

The model soil with 100 cm depth was prepared using Toyoura standard sand, by air-pluviating it into a large laminate shear box, and then by saturating it with de-aired water.

Fig.3 shows the soil-structure model and the arrangement of the sensors. The parts off from the piles in the transverse direction to the excitation were assumed as the free field.

The primary natural frequency of the soil was 18.0 Hz by microtremor measurement. The relative density of the soil was Dr=45% after the saturation.

The structure model simulated a 25-story reinforced concrete (RC) structure on RC piles. The height of the model was further contracted to approximately 1/4. The model was prepared using flat iron blocks corresponding to footing and upper 9 blocks with plate springs in between. The pile model was made by substituting equivalent 9 (3x3) piles. The piles were solid aluminum rods of a diameter of 50 mm. The primary, secondary, and tertiary natural frequencies of the model structure were 6.0 Hz, 15.6 Hz, and 24.4 Hz, respectively, with fixed footing. The damping ratio was in the range between 0.6 and

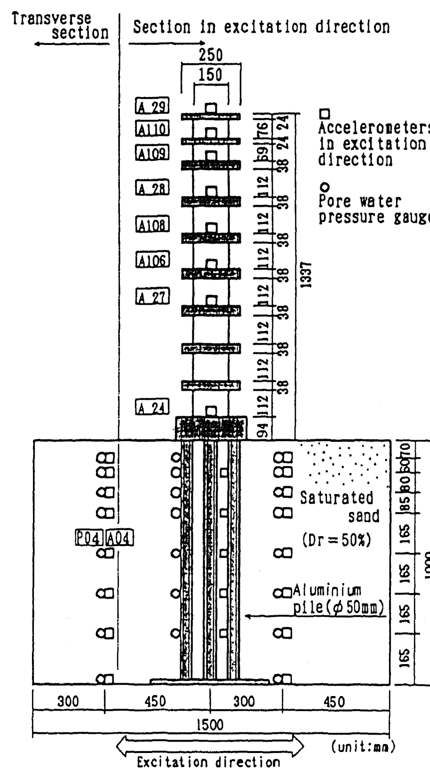

Fig.3 Model and arrangement of sensors in a shear vessel on shaking table

1.0%. The piles were rigidly fixed to the bottom of the vessel and the footing.

The NS component of the acceleration record obtained at the Akita Harbor during the Nihonkai Chubu Earthquake in 1983 was used for the excitation.

## 4. PARAMETERS FOR ANALYSIS MODEL

Table 1 shows the parameters for liquefaction analysis model. Most of them were determined according to laboratory tests, such as triaxial tests, simple shear tests, etc. [Mori et al.(1990)].

The upper structure and piles were modeled as a lumped mass-beam system. The axial deformation of the piles was taken into consideration as a rotating spring. These are all assumed to be linear elastic. The mass of the near field system and the interaction springs were calculated using Mindlin-II solution, following J. Penzien et al.(1972). The damping used in the analysis was Rayleigh damping, which was determined to be 1% at the primary and the secondary natural frequencies of the upper structure.

## 5. COMPARISON OF TEST AND ANALYSIS RESULTS

Fig.4 shows the time histories of major responses of soil and structure.

Table 1 Parameters for soil liquefaction analysis

| Material constants of soil | | | |
|---|---|---|---|
| Relative density | Dr | | 50 % |
| Shear modulus G | | ( 0~ 30 cm depth) | 309 $tf/m^2$ |
| | | (30~100 cm depth) | 1180 $tf/m^2$ |
| Internal friction angle | $\phi'$ | | 37 deg. |
| Coefficient of permeability | $k$ | | $1.0\times10^{-4}$ m/s |
| Coefficient of earth pressure at rest | $K_o$ | | 0.40 |
| Void ratio | $e$ | | 0.787 |
| Unit weight(saturated) | $\gamma_{sat}$ | | 1.92 $tf/m^3$ |
| **Parameters for stress path model** | | | |
| Coefficient of volume compressibility | $m_v$ | a | 6.398 |
| | | b | -0.579 |
| Internal friction angle for model | $\phi$ | | 24.3 deg. |
| Angle of phase transformation | $\theta_s$ | | 21.4 deg. |
| Parameters for pore water pressure | | Bp | 13.22 |
| | | Bu | 3.66 |

* $m_v = a\ (\sigma_{vo}')^{b}$

### 5.1 Response of soil

The excess pore pressure rose to the level which corresponds to the initial effective overburden pressure down to a depth of 100 cm in the tests and to a depth of 80 cm in the analyses.

The time histories of these rises are as follows: in the tests the pore pressure began

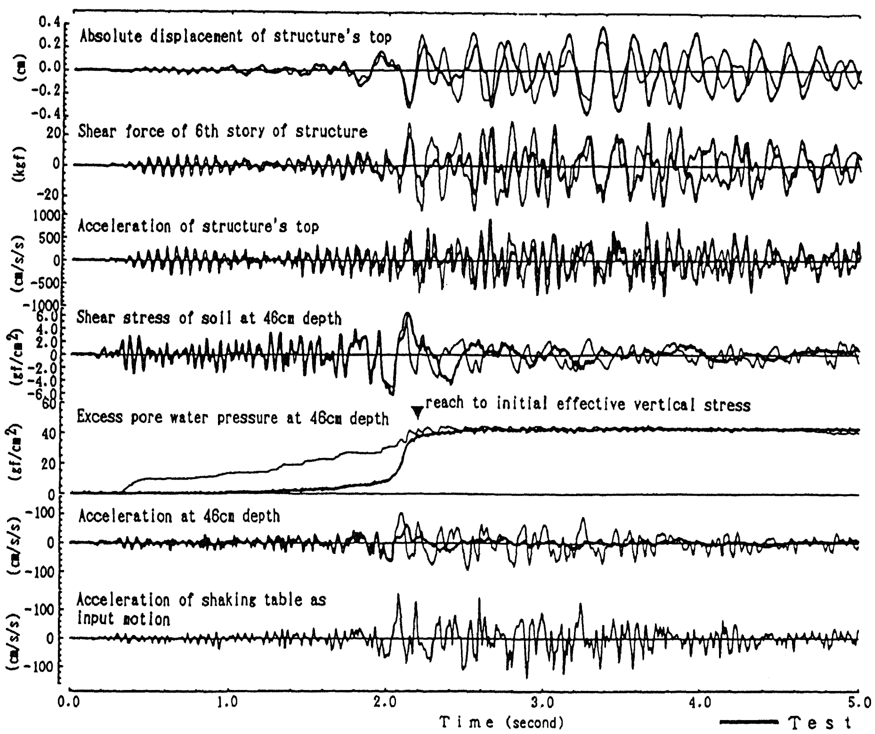


Fig.4 Time histories of response of soil and structure(Comparison between test and analysis)

to be generated at 0.9 sec., showed a sudden rise at around 2.0 sec., and reached the state of liquefaction at around 2.4 sec; in the analysis the pressure began to be generated at 0.3 sec. and then gradually rose to reach the state of liquefaction at 2.4 sec.. It is understood that under low confining pressure conditions as in these tests the pore pressure is more readily generated in the analyses than in the tests.

### 5.2 Response of structure on pile foundation

Before 1 sec. the responses of the acceleration and the shear force of the analysis were twice as great as those of the test. Between 1 sec. and 2 sec., when the pore pressure began to be generated in the test, every response of both the test and the analysis nearly conformed to each other. However, in the time zone between 2.1 sec. and 2.5 sec., when the pore pressure showed a sudden rise, considerable differences were observed between the responses of the analysis and the test.

After 2.5 sec., when liquefaction was in the steady state, the phases of both corresponded well, especially at around 3.5 sec. and 4.5 sec. when the long period elements predominant. Since the secondary natural frequency elements of the pile-structure were assessed as smaller in the analysis than those observed in the test, both were in the different phases when shorter period elements predominant. However, they were on the same level in terms of the amplitude.

Fig.5 shows the distribution of maximum values of the response acceleration and absolute displacement on the top of the upper structure and the shear force . In addition

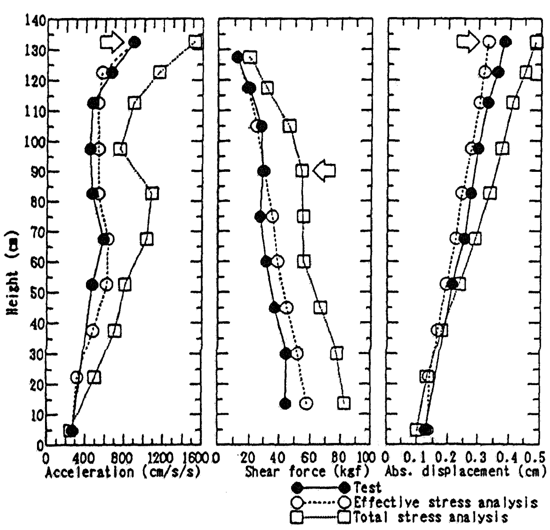


Fig.5 Distribution of maximum value of structure's response

to the results of the test and the analysis, the results of the total stress analysis are shown in the figure. No consideration is taken into the total stress analysis as to the influence of the pore pressure on the interaction springs.

Although the results of the total stress analysis showed greater response than those of the test at every part, the results of the effective stress analysis nearly conformed to the test results. It is considered that the effective stress analysis is effective for the design of structures constructed on the saturated loose sand deposits.

## 6. CONCLUSION

(1) Prediction of Soil Liquefaction

1 Under small shear stress as in these tests, the pore pressure was more readily generated in the analysis models than in the tests.

2 The analysis was able to predict the test results in terms of the time to reach liquefaction and the depth of liquefaction.

(2) Dynamic Response Characteristics of the Structure

1 In the time zone when the pore pressure suddenly rose, there were considerable differences between the tests and the analyses as for the response of the structure. Thus, the prediction was difficult.

2 When the pore pressure was small, and when liquefaction was in the steady state, the proposed analysis was able to simulate the test results.


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